QUANTIFYING THE RISK OF GEOTECHNICAL SITE INVESTIGATIONS

JASON SCOTT GOLDSWORTHY

B.E. (Hons), M.I.E. (Aust)

Thesis submitted for the Degree of
Doctor of Philosophy (PhD)

THE UNIVERSITY OF ADELAIDE AUSTRALIA

School of Civil and Environmental Engineering

JANUARY 2006

For my grandfather,
Kevin Marsh,
for his
dedication
and devotion

"people know the price of everything
and the value of nothing"

Oscar Wilde (1854-1900)

PREFACE

This thesis is the culmination of three and a half years work between July 2002 and January 2006. To the author's knowledge, all information and material obtained from other sources has been credited through citations and references. The following sections contain material for which the author claims originality.

In **Chapter 3**:

- Development and implementation of a method to investigate the risk and reliability of foundation designs based on the results from a site investigation.

In **Chapter 4:**

- Identification of a worst case scale of fluctuation (SOF) which is a function of the size of the averaging domain;
- Using a field translation technique to reduce aliasing or griding when generating three-dimensional random fields based on a lognormal distribution; and
- Use of a depth constraint to reduce the contribution of small strains on settlement estimates.

In **Chapter 5**:

- Measurement of the conservatism inherent in settlement prediction techniques for the analysis and design of a foundation on a soil with a spatially random elastic modulus; and
- Identification of an influence region within which an averaged elastic modulus value yields settlement estimates that accommodate soil variability.

In **Chapter 6**:

- Measurement of the effect of site investigations on the selection of design parameters;

- Analysis of the effect of site investigations on foundation design.

In **Chapter 7**:

- Reliability analysis of foundation designs based on the results from a site investigation in comparison with an optimal foundation design achieved using the complete knowledge of the soil;

- Use of an average design error to measure degree of under- and over-design of a foundation design based on the results from a site investigation;

- Recommendation of a single sampling location in a foundation system consisting of multiple footings; and

- Evaluation of the effect of measurement errors on the design of a foundation.

In **Chapter 8**:

- Risk assessment of a foundation designed on the basis of results from a site investigation;

- Identification of an optimal site investigation expenditure that yields a foundation design with lowest financial risk;

- Evaluation of the benefits of increased site investigation expenditure or sampling on the financial risk of a design;

- Identification of the most cost-effective types of site investigation tests.

In **Chapter 9**:

- Evaluation of the optimal site investigation strategy at three soil sites, where sufficient soil data has been made available for accurate characterisation of the soil variability.

The following publications have resulted from the research contained within this thesis:

Goldsworthy, J S, Jaksa, M B, Kaggwa, G W S., Fenton, G A, Griffiths, D V and Poulos, H G (2005). “Reliability of Site Investigations Using Different Reduction Techniques for Foundation Design,” *9th International Conference on Structural Safety and Reliability*, Rome, Italy, pp. 901–908. (On CD.)

Jaksa, M B, Goldsworthy, J S, Fenton, G A, Kaggwa, G W S, Griffiths, D V, Kuo, Y L and Poulos, H G (2005). "Towards Reliable and Effective Site Investigations," *Geotechnique,* 55(2), pp. 109-121.

Goldsworthy, J S, Jaksa, M B, Kaggwa, W S, Fenton, G A, Griffiths, D V and Poulos, H G (2004). "Cost of Foundation Failures Due to Limited Site Investigations," *The International Conference on Structural and Foundation Failures*, Singapore, pp. 398-409.

Goldsworthy, J S and Jaksa, M B (2004). "Effect of Design Models and Test Numbers on the Design of Pad Foundations," *6th Australian Young Geotechnical Professionals Conference*, Gold Cost, Australia, pp. 74-79.

Goldsworthy, J S, Jaksa, M B, Fenton, G A, Kaggwa, W S, Griffiths, D V, Poulos, H G and Kuo, Y L (2004). "Influence of Site Investigations on the Design of Pad Footings," *9th Australia New Zealand Conference on Geomechanics*, Auckland, New Zealand, pp. 282-288.

Abstract

The site investigation phase plays a vital role in any foundation design where inadequate characterisation of the subsurface conditions may lead to either a significantly over designed foundation that is not cost-effective, or an under-designed foundation, which may result in foundation failure. As such, the scope of an investigation should be dependent on the conditions at the site and the importance of the structure. However, it is common for the expense dedicated to the site investigation to be a fraction of the total cost of the project, and is typically determined by budget and time constraints, and the experience and judgement of the geotechnical engineer. However, additional site investigation expenditure or sampling is expected to reduce the financial risk of the design by reducing the uncertainties in the geotechnical system and protecting against possible foundation failures.

This research has quantified the relative benefits of undertaking site investigations of increased and differing scope. This has been achieved by simulating the design process to yield a foundation design based on the results of a site investigation. Such a design has been compared to an optimal design that utilises the complete knowledge of the soil, which has only been possible due to the use of simulated soils. Comparisons between these two design types indicate the performance of the site investigation to accurately or adequately characterise the site conditions. Furthermore, the design based on the results of the site investigation have been analysed using the complete knowledge of the soil. This yields a probability of failure and, therefore, has been included in a risk analysis where the costs associated with the site investigation have been measured against the financial risk of the design. As such, potential savings in financial risk for increased site investigation expenditure have been subsequently identified.

A Monte Carlo analysis has been used in this research to incorporate the uncertainties in the foundation design process. Uncertainties have been included due to soil variability; sampling errors; measurement and transformation model errors; and errors related to the use of a simplified foundation response prediction method. The Monte Carlo analysis has also provided the means to obtain results in a probabilistic framework to enable reliability and risk analyses. Computer code has been specifically developed with an aim to: generate a simulated soil that conforms to the variability of soil properties; simulate a site investigation to estimate data for a foundation design; simulate the design of a foundation and conduct a reliability and risk analysis of such a design.

Results indicate that there are significant benefits to be derived from increasing the scope of a site investigation in terms of the risk and reliability of the foundation design. However, it also appears that an optimal site investigation scope or expenditure exists where additional expenditure leads to a design with a higher financial risk due to the increased cost of the site investigation. The expected savings in terms of financial risk are significant when compared to the increased investigation cost. These results will assist geotechnical engineers in planning a site investigation in a more rational manner with knowledge of the associated risks.

STATEMENT OF ORIGINALITY

This thesis contains no material which has been accepted for the award of any other degree or diploma in any university, or other tertiary institution and, to the best of my knowledge and belief, contains no material previously published or written by another person, except where due reference has been made in the text.

I give consent to this copy of my thesis, when deposited in the University Library, being made available for loan and photocopying.

Signed: Date: 30 January 2006

ACKNOWLEDGMENTS

This document encapsulates the last three and half years of research I have undertaken at the University of Adelaide, Australia. This period has been a special part of my life that I have thoroughly enjoyed and will always look back on with fond memories. This is direct result of the support, friendship and direction that several people have afforded me. First and foremost, I owe a great deal to my principal supervisor, Dr Mark Jaksa from the University of Adelaide. He has not only provided a great source of direction and support throughout this research, but has also become a close friend. For this I am forever indebted to him. I would also like to share my appreciation for the support of my co-supervisor, Dr William Kaggwa, also from the University of Adelaide. Although his involvement in this research has not been to the same extent as Dr Jaksa's, he has provided invaluable direction regarding the significance and application of results and the final structure of this document.

Acknowledgement is also made to the Australian Research Council who funded this research as part of Discovery Project Grant. Without their financial assistance, this research would not have been possible.

I would also like to acknowledge three additional people who have all been directly involved in this project: Professor Vaughan Griffiths, from the Colorado School of Mines, USA; Professor Harry Poulos, from Coffey Geosciences and the University of Sydney in Australia; and Professor Gordon Fenton, from Dalhousie University in Canada. Professor Poulos has given valuable direction regarding the foundation design process and the geotechnical engineering industry in general. Professor Griffiths graciously provided the three-dimensional finite element analysis code that has been used for the optimal design. Furthermore, he also provided invaluable direction regarding the prediction of footing settlements using finite element analyses. I would also specially like to thank Professor Fenton, who not only offered the use of his random field generator, which enabled the generation of a simulated soil, but also spent nearly 12 months at the University of Adelaide assisting with this research. Thus, his contribution and influence on this research should not be underestimated and I would like to offer my sincere gratitude for his time and assistance.

I have also been afforded some advice from Professor Fred Kulhawy, from Cornell University in USA, and Associate Professor Kok Kwang Phoon, from the National University of Singapore. Professor Kulhawy provided general direction in the early stages of this research, while Associate Professor Phoon has provided direction in the latter stages. For this I thank them both very much and appreciate their valuable time.

The wide range of results shown in this research has required the use of significant computing resources at the University of Adelaide. Therefore, it has been necessary to program, compile and build the computer code for execution on several different computing systems and two different platforms. I would like to thank four people in particular for the assistance they have given me in this capacity: Dr Stephen Carr, from the School of Civil and Environmental Engineering at the University of Adelaide, who has not only helped with the code generation, but also several other issues regarding software and computing; and Mr Paul Coddington, Patrick Fitzhenry and Grant Ward, who all provided assistance with building and running the computer code on a supercomputer managed by the South Australia Partnership for Advanced Computing (SAPAC) called Hydra. These people have gone beyond the call of duty to lend assistance regarding the intricacies of running computer code on a multi-processor system.

Finally, I would also like to thank my close friends and family who have supported me and helped me to enjoy this journey. I would especially like to thank my parents, Robyn and Tony Goldsworthy, who have always encouraged me to achieve my goals and afforded me unconditional support. I would also like to acknowledge my grandparents, Pat and Kevin Marsh, who have been an amazing source of support and encouragement, not only for these last few years. Last by no means least, I would like to thank my partner, Natalie Pearce, who has endured this journey with me, whilst always keeping a smile on her face. She is an amazing person who has ensured that I have kept on track and never lost sight of the final goal. For this I am eternally grateful.

Last of all, I would like to thank you, the reader, for spending the time in ploughing through this research, which I hold very dear. I hope you enjoy the read and find it enlightening in some way.

CONTENTS

ADDENDUM

CHAPTER 1 – INTRODUCTION

No changes.

CHAPTER 2 – LITERATURE REVIEW

Page 17, Paragraph 1, add between Sentence 2 and 3:

Eq (2.5) is the common idealisation of the strain equation which, in actual fact, includes a radial strain component. However, for most foundation design applications, the strain estimate considers only the vertical component and, as such, will be adopted in this research.

Page 29, add after Paragraph 4:

The main reason for detrending data is to obtain a set of properties that are largely spatially independent (Fenton 199b). This is because classic statistical methods require independent and identically distributed data. The detrending process typically involves using regression analysis to fit a low-order polynomial (Jaksa et al. 1997) to the data set and removing it from the property value to leave a random residual, $w(z)$. However, Phoon et al. (2003) comments that detrending is not unique, and different procedures will result in different random residuals. As discussed earlier, Fenton (1999) warns that a trend should only be removed if it has physical meaning. Furthermore, Fenton (1999b) also warns that trends should be investigated with caution, because they could be a part of a larger process.

Phoon et al. (2003) suggests that the success of the detrending process can be measured by comparing the results of a statistical analysis on the random residual after a trend with an increasingly higher order polynomial is removed from the data. However, Fenton (1999) found that an apparent trend in a set of cone penetration test (CPT) data had little significance to the resulting statistical analysis. Therefore, Fenton (1999) did not remove the trend which would have resulted in a different mean, variability and correlation structure.

Although past research has demonstrated that it is important to attain a statistically homogeneous data set for a meaningful statistical analysis, care must be taken when detrending data, because the apparent trend may be a part of a much larger process that is not captured in a finite sample data set.

Page 31, Paragraph 2, Sentence 1:

The correlation between two properties is bounded by -1 and 1, where $\rho_\tau = \pm 1$ relates to the observations being perfectly correlated (either positively or negatively) and $\rho_\tau = 0$ relates to the observations being completely unrelated or purely random, provided that the observations [X and Y in Equation (2.19)] are not functionally related (Vanmarcke 1977a).

CHAPTER 3 – METHODOLOGY DEVELOPMENT

Page 62, Paragraph 1, Sentence 1:

As LAS appears to generate random fields considerably faster than the Turning Bands Method, and does not suffer from a symmetric covariance structure (Fenton 2002), like the Fast Fourier Transformation, it is adopted for this research.

Page 65, Equation 3.3, replace with:

$$\sigma_{\ln x} = \sqrt{\ln\left(1 + \frac{\sigma_x^2}{\mu_x^2}\right)} \tag{3.3}$$

Page 66, Paragraph 1, add at end:

It should also be recognised that the SOF used throughout this research is the SOF of the underlying Gaussian random field, and not the SOF of the lognormal random field.

Page 73, Paragraph 3, add after Sentence 2:

It should also be recognised that the reduction techniques only average properties in the same horizontal plane. In other words, the reduction techniques do not combine elastic moduli at different depths.

Page 74, Paragraph 2, add at end:

Furthermore, the DMT can also be undertaken at smaller than 1.5 m depth intervals. However, it is rare that 30 DMT tests are taken in one borehole or sampling location, and the DMT is typically more discrete than the CPT. Therefore, for the purposes of this research, a vertical sampling rate of 1.5 m for the DMT is adopted.

Page 99, Paragraph 1, delete Sentence 1.

Page 99, Paragraph 2, Sentence 2:

The analysis to investigate the convergence of the Monte Carlo analysis is based on a foundation system consisting of 9-pad footings, as shown in Figure 3-30, centred on a 50 m × 50 m site with a 30 m deep soil layer.

CHAPTER 4 – VERIFICATION OF METHODOLOGY

Page 110, Paragraph 4, add at end:

However, it should also be recognised that the Chi-square goodness-of-fit test is based on independent samples, and as the SOF increases, the soil properties become more correlated and less independent. Therefore, the Chi-square test is less applicable when the SOF is large.

Page 136, Paragraph 3, Sentence 2:

Since the elastic modulus, E_i, is stochastic and represented by a lognormal random variable, the settlement can also be expressed in terms of a lognormal stochastic variable.

Page 137, Equation 4.11, replace with:

$$\mathrm{Cov}[S_1, S_2] = \left(\frac{1+COV_E^2}{\mu_E}\right)^2 \left[\sum_{i=1}^{n}\sum_{j=1}^{n} C_i^{1**} C_j^{2**} \left(1+COV_E^2\right)^{\rho_{ij}} - \sum_{i=1}^{n} C_i^{1**} \sum_{j=1}^{n} C_j^{2**}\right] \quad \textbf{(4.11)}$$

CHAPTER 5 – EFFECT OF DIFFERENTIAL SETTLEMENT TECHNIQUES ON THE DESIGN AND ANALYSIS OF A PAD FOUNDATION

Page 158, add between Paragraphs 1 and 2:

The results shown in Figures 5-8 and 5-9 suggest that the variability of settlement estimates using 3DFEA is typically smaller than that from the other prediction models. This fact is evident because the distributions for 3DFEA settlement are narrower than the others. The 3DFEA settlement estimates are less variable because they make use of every soil property in the field, whereas the other prediction models only use a sample of elastic moduli to yield a settlement. Therefore, increased averaging occurs in 3DFEA settlement estimates, and therefore, the results are less variable. This explanation is also valid for comparisons between the other settlement prediction models. Comparisons between the variability are also influenced by the degree of conservatism in the model. It is shown later that more conservative prediction techniques, such as the Schmertmann Modified, yield more variable results. This is because the settlement estimates are closely linked to a lognormal distribution, and the variance and mean are related.

Page 162, add between Paragraphs 3 and 4:

Equation (5.7) assumes that the settlement estimates given by 3DFEA and the other techniques are independent. Therefore, the probabilities given in Table 5-3 also assume that the settlement estimates are independent. In actual fact, the settlement estimates will have some correlation because the same elastic moduli have been used in predicting the settlement. However, to keep this form of analysis relatively straightforward, the settlement estimates are assumed to be independent. The numerical analysis described later in this chapter, where footing settlement is determined as part of a Monte Carlo simulation incorporates the correlation between 3DFEA settlements and those from the other prediction techniques.

Page 188, add after Paragraph 3:

It should also be noted that the probabilities of under- and over-design, shown in Figures 5-30(a) and (b) respectively, do not add to unity for the same soil SOF and COV combination. This is because there is also a probability that the design based on 3DFEA and Schmertmann 2B-0.6 settlement prediction techniques will be equal. This is the case of the Schmertmann method yielding an optimal design, as is discussed later. It is also introduced, in later chapters, that the probability of obtaining an optimal design is relatively high because of the discretisation in footing size to keep 3DFEA computational times manageable.

CHAPTER 6 – EFFECT OF SITE INVESTIGATIONS ON DESIGN PARAMETERS AND THE DESIGN OF A PAD FOUNDATION

Page 198, Paragraph 3, Sentence 2:

A sampling location includes all elastic moduli in a vertical sample, leading to 60 values spaced at 0.5 m intervals.

Page 198, Paragraph 3, add between Sentence 5 and 6:

In all cases, the target mean of the random field is set to be 10,000 kPa.

Page 204, Paragraph 2, Sentence 10, replace:

This is further discussed later in this section.

with

Although this may seem counterintuitive, the average design parameter is influenced by the variability of the sampled elastic modulus values. For example, in the limiting case when only a single elastic modulus is sampled, the variance in the sample is zero. However, when additional samples are taken, the variability increases and tends toward the target variance. The increase in average design parameter is further influenced by the lognormal distribution, where a higher variance results in an increasing mean. Therefore, the results in Figure 6-4, which show that the parameter variability increases as the sampling effort grows, also causes a larger average design parameter.

Page 209, Paragraph 1, add at end:

However, in this case, the ID and I2 methods yield identical results. Therefore, the ID result is not shown in Figures 6-8 and 6-9.

Page 219, add between Paragraphs 3 and 4:

The recommendation of an absolute number of sampling locations over a sampling rate may at first seem counterintuitive; however, the influence of property variability must be carefully considered. It seems from the results shown in Figures 6-15 and 6-16 that 5 sampling locations yields the best answer. This is most likely because adequate averaging between soil properties occurs with this number of sampling locations, and the property variability is, as a result, low. However, if the site size is reduced a sampling rate yields a recommendation for fewer sampling locations, which results in less property averaging.

Therefore, the property variability is high, and the error in the design is large. On the other hand, if the site is larger, a sampling rate infers a greater number of sampling locations, which may be redundant and has little influence on the accuracy of the design.

Hence, it follows that sampling methods that retain more information per sampling location achieve more accurate designs with fewer sampling locations. For example, the cone penetration test (CPT), which is a relatively continuous sampling method, requires fewer sampling locations to achieve an equally accurate design, than the standard penetration test (SPT). This type of analysis is discussed later, yet, in this research, the CPT is modelled to attain only 3 times as many samples as the SPT (§3.4.3). It should also be remembered that different test types are also influenced by their own inherent errors, as discussed earlier in Chapter 2 (§2.3.4), which have a considerable impact on the accuracy of the design. The influence of such inherent testing errors is also discussed later.

Although the results shown in Figures 6-15 and 6-16 suggest the use of an absolute number of sampling locations is preferred, it is important to remember that this analysis is based on a single-layered soil deposit that is statistically homogeneous. Additional sampling is always recommended for soil deposits with multiple layers and geological anomalies.

CHAPTER 7 – RELIABILITY ASSESSMENT OF A SITE INVESTIGATION IN TERMS OF FOUNDATION DESIGN

Page 290, Paragraph 1, add at end:

Figure 7-24 shows that the average design error is low when the soil SOF is small. This same phenomenon was shown in Figure 7-23(a) where the average design error was relatively small when a sample location was positioned around the site. The average design error is relatively low when the soil SOF is small because of two reasons. Firstly, the apparent variability of the soil properties is less when the soil SOF is small, as shown in Chapter 4 (§4.2.1). Secondly, and more importantly, considerable local averaging occurs when the soil SOF is small. Therefore, the variability of the sampled properties used in the footing design is less and, as a result, the average design error is low. In the limiting case, where the soil SOF approaches zero, soil properties at 0.5 m spacings will be the same because of local averaging. In this case, the average design error is zero.

Chapter 8 – Risk Assessment of a Site Investigation in Terms of Foundation Design

Page 310, Paragraph 3, Sentence 2:

Figures 8-6 and 8-7, as well as other figures in Chapter 8, the terminology of "failure cost", or "cost", is used for the vertical axis label. These costs are, in fact, expected costs and reflect the average calculated over 1000 Monte Carlo realizations. Also note the different scales on the vertical axis for total and failure costs.

Page 347, Paragraph 4, Sentence 3:

Furthermore, it is also important to consider that this analysis considers only a single, statistically homogeneous layer of soil.

Chapter 9 – Analysis Using Specific Soil Data

No changes.

Chapter 10 – Summary and Recommendations

No changes.

References

Fenton G A. (1999b). "Estimation for Stochastic Soil Models." *Journal of Geotechnical and Geoenvironmental Engineering*, 125(6), pp. 470-485.

Phoon K K, Quek S T and An P. (2003). "Identification of Statistically Homogeneous Soil Layers Using Modified Bartlett Statistics." *Journal of Geotechnical and Geoenvironmental Engineering*, 129(7), pp. 649-659.

LIST OF FIGURES

Chapter 3 Methodology Development

Chapter 4 Verification of Methodology

Chapter 5 Effect of Different Settlement Prediction Techniques on the Design and Analysis of a Pad Foundation

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Chapter 8 Risk Assessment of Site Investigations in Terms of Foundation Design

Chapter 9 Analysis Using Specific Soil Data

Chapter 10 Summary and Recommendations

Appendices

Appendix A Settlement Analysis

Appendix B Average and Variance of Foundation Design Results

Appendix E Foundation Design Costs

List of Tables

Chapter 8 Risk Assessment of Site Investigations in Terms of Foundation Design

Chapter 9 Analysis Using Specific Soil Data

Chapter 10 Summary and Recommendations

NOTATION

1Q	1st quartile selection
2:1	2:1 settlement prediction technique
3DFEA	Three dimensional finite element analysis
A	Area – subscripts used to denote object (e.g. footing, influence area)
A_{opt}	Optimal footing area designed using complete knowledge
B	Shape factor for settlement estimate
a	Shape factor for settlement estimate
b	Width or least plan dimension of the footing
b_{infreg}	Width of the influence region
b'	Effective width or least plan dimension of the footing
C^*	Variable representing correction factors, load and footing size
C^{**}	Variable representing strain influence factor and C^*
C_1	Footing embedment correction factor (Schmertmann)
C_2	Time correction factor (Schmertmann)
CK	Complete knowledge of the soil (all properties known)
COV	Coefficient of variation
Cov[.]	Covariance operator
CPT	Cone penetration test
c	Cohesion
c_w	Width of column
D	Depth of the compressible soil layer
D_A	Test parameter A from DMT
D_B	Test parameter B from DMT
DE	Design error
d_e	Footing embedment depth
DFT	Discrete Fourier transformation method

DMT	Marchetti flat plate dilatometer test
D_r	Relative density
D_v	Averaging domain
d	Thickness of footing for beam shear
d_{om}	Depth to top of reinforcing
d_t	Total thickness of footing
E	Elastic modulus
E[.]	Expectation operator
E_{ave}	Averaged elastic modulus value
E_{COV}	Coefficient of variation of elastic modulus field
E_D	Elastic modulus from DMT
E_f	Elastic modulus value taken directly from random field
E_{PMT}	Elastic modulus from PMT
E_r	Resultant elastic modulus value after effects of system uncertainty
E_{SOF}	Scale of Fluctuation of elastic modulus field (isotropic)
e_i	Proportion of element size in relation to total
F	Random variable representing load
FFT	Fast Fourier transformation
FORM	First order reliability method
FOS	Factor of safety
FOSM	First order second moment reliability method
f'_c	Yield strength of concrete
f_{cv}	Shear capacity of concrete
f_i	Proportion of number of samples in relation to number of elements
f_s	Sleeve friction from CPT
G	Shear strain modulus
GA	Geometric average
H	Depth of stress change
HA	Harmonic average
H_b	Height of structure of building
h	Difference between corner and middle settlements of a flexible footing
I_1; I_2; I_F	Shape factors
I2	Inverse distance squared weighted
ID	Inversed distance weighted
I_p	Influence factor

Ip_x; Ip_y	Size of the site investigation in x- and y-directions
I_z	Strain influence factor
Jan	Janbu settlement prediction technique
K_D	From DMT
k	Bulk modulus
k_s	Modulus of subgrade reaction
L	Shape factor for settlement estimate
LAS	Local average subdivision
LI	Liquid index
l	Length or largest plan dimension of the footing
M	Shape factor for settlement estimate
MA	Moving average method
MN	Minimum value selection
MOS	Margin of safety
MPI	Message passing interface (parallel processing)
m	Random variable representing measurement error
m_b	Random variable representing bias component of measurement error
m_F	Mean of random variable representing load
m_R	Mean of random variable representing capacity
m_r	Random variable representing random component of measurement error
N	SPT blow count
N_γ	Bearing capacity factor
N_0	Shape factor for settlement estimate
N_c	Bearing capacity factor
New	Newmark settlement prediction technique
N_q	Bearing capacity factor
n	Number of samples to reduce
n_f	Number of footings in foundation system
n_l	Number of discretised layers
n_r	Number of realisations
n_t	Total population size
OCR	Over-consolidation ratio
P	Applied footing load
Per	Perloff settlement prediction technique

PI	Plastic index
PMT	Pressuremeter test
p_f	Probability of failure
p_L	Pressuremeter limit stress
p_{od}	Probability of over design
p_{op}	Probability of attaining an optimal design
p_{ud}	Probability of under design
Q_{su}	Side resistance
Q_{tu}	Tip resistance
Q_u	Available capacity
Q_{ud}	Design uplift capacity
q	Applied footing pressure
q_a	Allowable bearing capacity
$q_{a_{net}}$	Net allowable bearing capacity
q_{av}	Averaging pressure over the footing contact area
q_c	Cone tip resistance from CPT
q_u	Unconfined compression strength
q_{ult}	Ultimate bearing capacity
q_z	Stress at depth z
q_z	Stress in soil at depth z
Δq	Change in stress
R	Random variability representing capacity
RFEM	Random finite element method
RG	Regular grid sampling pattern
RN	Simple random arrangement of sample locations
r_i	Internal radius of annulus
r_o	External radius of annulus
S	Estimated footing settlement
SA	Standard arithmetic average
Sch2B	Schmertmann settlement prediction technique based on 2B-0.6 strain distribution
SchM	Schmertmann settlement prediction technique based on Modified strain distribution
SE	Settlement error
SFEM	Stochastic finite element method
SGS	Sequential Gaussian simulation

SI	Site investigation knowledge of the soil (based on sampling)
SI_{opt}	Optimal site investigation cost, based on yielding the lowest total cost
SI^{*}_{opt}	Optimal site investigation cost for the worst case SOF
SIS	Sequential indicator simulation
SOF	Scale of fluctuation (measure of correlation distance)
SOSM	Second-order second-moment reliability method
SPT	Standard penetration test
SR	Stratified random sampling pattern
St_x; St_y	Size of the site in x- and y-directions
s_γ	Bearing capacity correction factor for unit weight
s_c	Bearing capacity correction factor for cohesion
s_i	Distance separating the *i*th sample location and the footing
s_F	Standard deviation of random variable representing load
s_R	Standard deviation of random variable representing capacity
s_{tot}	Total distance separating all sample locations and the footing
s_u	Undrained shear strength
sv	Random variable representing uncertainty due to spatial variability
T	Transformation model
T&G	Timoshenko and Goodier settlement prediction technique
TBM	Turning bands method
TBM	Turning bands method
TT	Triaxial test
$t(z)$	Trend value at depth z
tm	random variable representing transformation model error
V	Shape factor for settlement estimate
V^*	Applied shear load on footing
V_1	Shape factor for settlement estimate
V_1^*; V_2^*	Components of the applied shear load, V^*
VST	Vane shear test
Var[.]	Variance operator
V_{u0}	Punching shear capacity
V_{uc}	Beam shear capacity
W	Weight of shaft
W_b	Width of structure or building
Wst	Westergaard settlement prediction technique

$w(z)$	Fluctuating residual value at depth z
w_L	Water content at liquid limit
w_n	Natural water content
w_p	Water content at plastic limit
X	Random variable
$X_{\ln}$	Lognormal random variable
x	Individual property that conforms to X
x_c	X coordinate
x_d	Distance to parabolic centroid
Y	Random variable
y_c	Y coordinate
Z	Centre-to-centre spacing of two adjacent footings
z	Depth in soil layer
Δz	Soil layer thickness
α	Shape factor for settlement estimate
β	Reliability index
β_1	Shape factor for beam shear capacity
β_τ	Covariance at lag τ
δ	Footing settlement
$\delta_{1\|2}$	Settlement of Footing 1 due to Footing 2
δ_{ann}	Settlement of rigid annulus
δ_c	Corner settlement of flexible footing
δ_m	Centre settlement of flexible footing
δ_r	Rigid footing settlement
ε	Strain
γ	Unit weight of soil
γ'	Effective unit weight of soil
γ_d	Dry unit weight of soil
$\gamma(D)$	Variance reduction based on an averaging domain of D
γ_h	Semivariogram value at distance h
η_0	Footing embedment correction factor
η_1	Layer depth correction factor
η_a; η_b	Fitted constants for the Janbu settlement relationship
κ	Deterministic coefficient representing settlement prediction technique coefficients and design criteria limits

λ	Differential settlement ratio $\lvert\delta_1-\delta_2\rvert/Z$
$\mu_{\ln x}$	Sample mean of the logarithm of x
μ_x	Sample mean of x
ν	Poisson's ratio
θ	Isotropic scale of fluctuation, SOF
θ_h	Vertical scale of fluctuation, SOF
θ_v	Vertical scale of fluctuation, SOF
θ_{wc}	Worst case scale of fluctuation, SOF
ρ_{min}	Percentage of steel reinforcing
ρ_τ	Correlation at lag τ
σ_e^2	Variance due to equipment effects
$\sigma_{\ln x}$	Sample standard deviation of the logarithm of x
$\sigma_{\ln x}^2$	Sample variance of the logarithm of x
σ_m^2	Variance due to measurement error
σ_n'	Overburden pressure
$\sigma_{p/o}^2$	Variance due to procedural and operator effects
σ_{sv}^2	Variance due to spatial variability
σ_T^2	Variance due to all forms of uncertainty
σ_{tm}^2	Variance due to transformation model error
σ_r^2	Variance due to random test effects
σ_x	Sample standard deviation of x
σ_x^2	Sample variance of x
ϕ	Angle of internal friction
ϕ_{red}	Concrete strength reduction factor
τ	Lag or separation distance vector = $\{\tau_x,\ \tau_y,\ \tau_z\}$
τ_x	Lag or separation distance in the x-direction
τ_y	Lag or separation distance in the y-direction
τ_z	Lag or separation distance in the z-direction
ω	Angle representing the rate of stress decrease
ψ°	Angle in degrees used to proportion footing area in relation to annulus
ψ^c	Angle in radians used to proportion footing area in relation to annulus
ξ_d	Design parameter
ξ_m	Measured soil property
$\xi(z)$	In situ soil property at depth z